Influence of soil–structure interaction on the response of seismically isolated cable-stayed bridge

B.B. Soneji*, R.S. Jangid

Department of Civil Engineering, Indian Institute of Technology Bombay, Mumbai 400 076, India

Abstract

Soil conditions have a great deal to do with damage to structures during earthquakes. This paper attempts to assess the influence of dynamic soil–structure interaction (SSI) on the behavior of seismically isolated cable-stayed bridge supported on a rigidly capped vertical pile groups, which pass through moderately deep, layered soil overlying rigid bedrock. In the present approach, piles closely grouped together beneath the towers are viewed as a single equivalent upright beam. The soil–pile interaction is idealized as a beam on nonlinear Winkler foundation using continuously distributed hysteretic springs and viscous dashpots placed in parallel. The hysteretic behavior of soil springs is idealized using Bouc–Wen model. The cable-stayed bridge is isolated by using high-damping rubber bearings and the effects of SSI are investigated by performing seismic analysis in time domain using direct integration method. The seismic response of the isolated cable-stayed bridge with SSI is obtained under bi-directional earthquake excitations (i.e. two horizontal components acting simultaneously) considering different soil flexibilities. The emphasis has been placed on assessing the significance of nonlinear behavior of soil that affects the response of the system and identify the circumstances under which it is necessary to include the SSI effects in the design of seismically isolated bridges. It is observed that the soil surrounding the piles has significant effects on the response of the isolated bridge and the bearing displacements may be underestimated if the SSI effects are ignored. Inclusion of SSI is found essential for effective design of seismically isolated cable-stayed bridge, specifically when the towers are very much rigid and the soil condition is soft to medium. Further, it is found that the linear soil model does not lead to accurate prediction of tower base shear response, and nonlinear soil modeling is essential to reflect dynamic behavior of the soil–pile system properly.

Keywords: Cable-stayed bridge; Soil–structure interaction; Pile group; Nonlinear springs; Earthquake; Isolation system

1. Introduction

Significant damage of bridges due to partial or complete collapse of piers has been observed in every major seismic event. The 1994 Northridge earthquake, 1995 Kobe earthquake and 2001 Gujarat earthquake have demonstrated that the strength alone would not be sufficient for the safety of bridges during the earthquake. For the past several years, the research is focused on finding out more rational and substantiated solutions for protection of bridges from severe earthquake attack. Seismic isolation is a strategy that attempts to reduce the seismic forces to or near the elastic capacity of the member, thereby eliminating or reducing inelastic deformations. The main concept in isolation is to reduce the fundamental frequency of structural vibration to a value lower than the predominant energy-containing frequencies of earthquake. The other purpose of an isolation system is to provide means of energy dissipation, which dissipates the seismic energy transmitted to the system. The isolation device, which replaces conventional bridge bearings, decouples the bridge deck (which alone is responsible for majority of the tower base shear) from bridge substructure during earthquakes thereby significantly reducing the deck acceleration and consequently the forces transmitted to the towers or piers.

Though the technique of seismic isolation has been successfully implemented world wide for buildings and short to medium span highway bridges [1–4], very limited
amount of work has been done on the use of seismic isolation for seismic response control of cable-stayed bridges. Ali and Abdel-Ghaffar [5] investigated the effectiveness of elastomeric (elastic and hysteretic type both) bearings for seismic isolation of cable-stayed bridges. Wesolowsky and Wilson [6] examined the efficacy of using seismic isolation to favorably influence the seismic response of cable-stayed bridges subjected to near-field earthquake ground motions. It is to be noted that in these studies on isolated cable-stayed bridges, the foundation of the bridge towers is assumed as rigid (embedded in solid rock) and there has not been any attempt to investigate the effects of SSI on the response of the bridges.

In recent years, several cable-stayed bridges have been constructed on relatively soft ground, which results in a great demand to evaluate the effects of soil–structure interaction (SSI) on the seismic behavior of the bridges, and properly reflect it in their seismic design. During these years, some studies have been conducted to comprehend the effects of SSI on the seismic behavior of non-isolated bridges [7–11]. Also, the studies have been carried out to investigate the seismic response of isolated reinforced cement concrete bridges considering SSI [12–14]. These studies have demonstrated that SSI generally tends to prolong the natural periods of bridge–foundation–soil systems, and may significantly affect the internal forces in structural members and displacement response of bridges. It has also been recognized that the way in which SSI affects the seismic behavior of bridges depends on the conditions of the bridge–foundation–soil system [15], suggesting a necessity to perform many detailed case studies.

In this paper, influence of dynamic SSI on the seismic response of cable-stayed bridge isolated by elastomeric bearings is investigated. The soil–pile interaction is idealized as a beam on nonlinear Winkler foundation (BNWF) using continuously distributed nonlinear springs and dashpots. The specific objectives of the study are: (i) to investigate the influence of flexibility of layered soil on the response of seismically isolated cable-stayed bridge; (ii) to assess the significance of nonlinearity in soil on the seismic response of the isolated bridge by comparing the results of SSI considering and ignoring nonlinearity and (iii) to identify the circumstances under which it is necessary to include the SSI effects in the design of seismically isolated cable-stayed bridges.

2. The cable-stayed bridge model

The bridge model used in this study is the Quincy Bay-View Bridge crossing the Mississippi River at Quincy, IL.
The bridge consists of two H-shaped concrete towers, double plane semi-harp-type cables and a composite concrete-steel girder bridge deck. The detailed description of the bridge is given in Wilson and Gravelle [16]. Here, a simplified lumped mass finite element model of the bridge developed for the investigation is as shown in Fig. 1(a). The finite element model of the towers is separately shown in Fig. 1(b). There are 28 cable members, 14 supporting the main span and 7 supporting each side spans. The cable members are spaced at 2.75 m c/c at the upper part of the towers and equally spaced at the deck level on the side as well as main spans. The relevant properties of the bridge deck (for equivalent steel area) and towers are given in Table 1 while those of the cables are given in Table 2. The bridge deck is assumed to be a continuous beam rigidly connected to the towers such that the deck moment will not be transferred to the tower through the deck–tower connection. For implementing seismic isolation, the isolation bearings are placed at each four supports of the deck replacing conventional bearings (refer Fig. 1(c) and (d)). The bridge towers are supported on rigidly capped vertical pile groups passing through moderately deep, layered soil overlying rigid bedrock. When piles are closely grouped together, the piles and soil work like a single rigid unit, and the problem becomes of the group working like a large pier [17,18]. Hence in present approach, piles closely grouped together beneath the towers are viewed as a single equivalent upright beam with second moment of area about \(x-x\)-, \(y-y\)- and \(z-z\)-axis equal to 82.72, 1057.726 and 189.74 m\(^4\); respectively, and cross-sectional area equal to 20.49 m\(^2\). The piles are spaced at three pile diameters, and the properties of the single equivalent beam include group effects.

It is worth mentioning that a cable-stayed bridge, in the rigorous sense, behaves nonlinearly when loaded. However, the research on ambient vibration survey of the cable-stayed bridge carried out by Wilson and Liu [19] has demonstrated that a completely linear model was sufficient to get reasonably accurate results for the Quincy Bay-View Bridge. Hence, no attempts are made to introduce nonlinearity into the model in this investigation.

Regarding modeling of the bridge components, the deck and the tower members are modeled as space frame elements. The cables are modeled as linear elastic space truss elements. The stiffness characteristics of an inclined cable can exhibit a nonlinear behavior caused by cable sag. This nonlinear behavior can be taken into account by linearization of the cable stiffness using an equivalent modulus of elasticity that is less than the true material modulus [20]. For the analysis of the bridge under consideration, Wilson and Gravelle [16] found the value of equivalent modulus essentially equal to the true modulus of elasticity. Hence, here nonlinearity due to cable sag is neglected and cables are treated as having a completely linear force–deformation relationship described by the true material modulus of elasticity. Moreover, cables are assumed to be capable of bearing tension as well as compression assuming that the pretension in the cables will take care of the compression induced in the dynamic analysis.

### 3. Modeling of the soil–pile system

In the last two decades, several numerical and analytical methods have been developed to compute the dynamic stiffness and seismic response factors of pile foundations accounting for soil–pile interaction. Most of these methods assumed linear viscoelastic response of the surrounding soil. Nevertheless, under moderate and strong seismic loading, pile foundations undergo significant displacements and the behavior of the soil-pile system can be nonlinear.

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#### Table 1

Properties of the deck and the towers of the cable-stayed bridge

<table>
<thead>
<tr>
<th>Part of the structure</th>
<th>Cross-sectional area (m(^2))</th>
<th>Moment of inertia about (z-z)-axis (m(^4))</th>
<th>Moment of inertia about (y-y)-axis (m(^4))</th>
<th>Moment of inertia about (x-x)-axis (m(^4))</th>
<th>Young's modulus (MPa)</th>
<th>Mass density (kg/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>0.827</td>
<td>0.341</td>
<td>19.760</td>
<td>0.027</td>
<td>205,000</td>
<td>7850</td>
</tr>
<tr>
<td>Tower part 1</td>
<td>14.120</td>
<td>28.050</td>
<td>531.580</td>
<td>15.390</td>
<td>30,787</td>
<td>2400</td>
</tr>
<tr>
<td>Tower part 2</td>
<td>14.120</td>
<td>28.050</td>
<td>670.970</td>
<td>15.390</td>
<td>30,787</td>
<td>2400</td>
</tr>
<tr>
<td>Tower part 3</td>
<td>17.540</td>
<td>30.620</td>
<td>1239.400</td>
<td>19.760</td>
<td>30,787</td>
<td>2400</td>
</tr>
<tr>
<td>Tower part 4</td>
<td>35.390</td>
<td>32.750</td>
<td>1422.420</td>
<td>27.640</td>
<td>30,787</td>
<td>2400</td>
</tr>
</tbody>
</table>

#### Table 2

Properties for the stay cables of the cable-stayed bridge

<table>
<thead>
<tr>
<th>Cable no.</th>
<th>Cross-sectional area (m(^2))</th>
<th>Young's modulus (MPa)</th>
<th>Cable weight (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0180</td>
<td>205,000</td>
<td>1765.80</td>
</tr>
<tr>
<td>2</td>
<td>0.0135</td>
<td>205,000</td>
<td>1324.35</td>
</tr>
<tr>
<td>3</td>
<td>0.0107</td>
<td>205,000</td>
<td>1049.67</td>
</tr>
<tr>
<td>4</td>
<td>0.0070</td>
<td>205,000</td>
<td>686.70</td>
</tr>
</tbody>
</table>
Studies on the nonlinear dynamic response of piles have been conducted with the finite element method [21]. The finite element method, which requires discretization of the pile and the surrounding soil, although powerful, is computationally very expensive. In contrast, the BNWF model is a versatile and economical approach. Trochanis et al. [22] have carried out the most comprehensive study on the nonlinear response of piles using a BNWF model utilizing a model of viscoplasticity, better known as the Bouc–Wen model [23,24], to describe the force–displacement relation of distributed nonlinear springs that approximate the soil reaction on the pile. Their study concentrated on the static and quasi-static (zero-frequency limit) loading of piles, and for this type of loading it was shown that the Bouc–Wen model predicts well the response of a variety of soil–pile systems.

Herein, the soil–pile interaction is idealized as a BNWF using continuously distributed nonlinear springs and viscous dashpots placed in parallel (Fig. 2(a)). The presence of the damper makes the model very efficient for the prediction of the pile response under dynamic loads since it accounts realistically for the energy that radiates outward. The response of the superstructure is investigated under three different types of soil surrounding the pile foundation, namely, soft, medium, and firm. The soils are considered in layers of different thickness resting on rock (Fig. 2(b)), with increasing stiffness and decreasing damping with depth. Assuming that the rigid bedrock is available at a depth of 25 m, soil springs are distributed at 2.5 m centers. Thus, the discretization of pile into 11 segments by using 10 springs is enough to achieve sufficient accuracy in the analysis. The spring coefficients have been computed by method suggested in Specification for Highway Bridges issued by Japan Road Association [25]. In the suggested method, reference soil reaction coefficient $k_0$ is first computed using Eq. (1), where $E_s$ is Young’s modulus of soil. The soil reaction coefficient per unit area, $k_r$, is obtained using Eq. (2), where $B_e$ is the width of the foundation perpendicular to the direction considered:

$$k_0 = \frac{1.2E_s}{30}, \quad (1)$$

$$k_r = k_0 \sqrt[3]{\frac{B_e}{30}}, \quad (2)$$

Young’s modulus can be evaluated using the relation

$$E_s = 2G_s(1 + \mu_s), \quad (3)$$

where $G_s$ is the shear modulus and $\mu_s$ is Poisson’s ratio for the soil.

It should be mentioned that, when using Eqs. (1) and (2), the units of $B_e$ and $E_s$ must be in centimeters and kg/cm$^2$, respectively. The horizontal spring coefficients ($k_s$) for each part of foundation are obtained by multiplying $k_r$ by the area of its surface perpendicular to the excitation direction. The horizontal spring coefficients are evaluated by taking the widths of foundation perpendicular to the longitudinal and transverse directions of the bridge as 21 and 9 m, respectively. Although it has been recognized that spring coefficients are frequency dependent, the spring coefficients computed using the method suggested in Japan Road Association [25] are frequency independent for practical use.

The second key parameter for soil is damping. Two fundamentally different damping phenomena are associated with soil, namely material damping and radiation.

Fig. 2. Schematic of dynamic BNWF model in layered soil strata. (a) BNWF model and (b) layered soil.
damping. Material damping can be thought of as a measure of the loss of vibration energy resulting primarily from hysteresis in the soil. The radiation damping is a measure of energy loss from the structure through radiation of waves away from the foundation. Expressions for damping coefficients are available in literature \[26,27\], on the basis of which the following simple approximation is derived \[28\]:

\[
c_s = Q_s \omega_d^{1/4} \rho_s V_s d + 2 \xi_s \frac{k_s}{\omega_k},
\]

where \(\xi_s\) and \(\rho_s\) are the damping ratio and mass density of the soil, \(d\) is the pile diameter, shear wave velocity, \(V_s = \sqrt{G_s/\rho_s}\), \(\omega_k = \pi V_s/2d\), \(d\) is the thickness of soil layer and \(d_0 = \sqrt{\omega_k d}/V_s\). The coefficient \(Q_s\) is given by the expression \(Q_s = 2 \left[1 + \frac{3.4}{\pi(1-\mu)}\right]^{1.25} \left(\frac{\pi}{4}\right)^{0.75}\).

The dynamic properties of the soils such as shear modulus, mass density, Poisson’s ratio, damping ratio, shear strength, etc. that vary with the depth are given in Table 3 \[29,30\].

The force resulting from the nonlinear spring alone is given by

\[
F_s = 2k_s q_0 + (1 - 1)k_s q Z,
\]

where \(q_0\) is the pile deflection at the location of the spring, \(x\) is a parameter that controls the post-yielding stiffness, \(q\) is the value of pile deflection that initiates yielding in the spring and \(Z\) is a hysteretic dimensionless quantity that is governed by the following equation:

\[
q Z = A q_0 + \beta q_0 |Z| Z^\gamma - \gamma q_0 Z^\gamma.
\]

In the above equation \(\beta\) is a parameter that controls the shape of the hysteretic loop. The hysteretic model of Eq. (7) was originally proposed by Bouc [23] for \(n = 1\), and subsequently extended by Wen [24], and used in random vibration studies of inelastic systems. It can be easily shown that when \(A = 1\) in Eq. (7), parameter \(k_s\) in (6) becomes the small-amplitude elastic distributed stiffness, and parameter \(x\) becomes the ratio of the post- to pre-yielding stiffness \[31\]. Based on this observation, all the subsequent analysis is carried out taking \(A\) and \(n\) equal to 1. The other parameters, which do not affect much the peak response, are kept constant (i.e. \(\beta = \gamma = 0.5\)). The plastic soil behavior (no hardening) at large pile deflections indicated that the ultimate postyielding stiffness of the soil is zero and, therefore, the parameter \(x\) in Eq. (6) is set equal to zero.

The value of \(q\) (pile deflection at which yielding initiates in the soil spring) in Eq. (7) at depth \(h\) is provided by

\[
q(h) = \lambda(h) \frac{\tau(h)d}{k_s/S},
\]

where \(S\) is the spacing between two adjacent springs, \(\tau(h)\) is the shear strength of the soil at depth \(h\) and \(\lambda(h)\) is a dimensionless quantity. Considerable research has been done to quantify \(\lambda\) \[32,33\]. \(\lambda\) values between 9 and 12 may be appropriate at depths where plane strain conditions are valid. At shallow depths, plane strain conditions are not valid due to vertical deformation of the soil during lateral motion of the pile. Hence, \(\lambda\) values of 2 or 3 have been suggested. The following expression for the variation of \(\lambda\) with depth is recommended \[33\]:

\[\lambda(h) = 3 + \frac{\sigma_z}{\tau(h)} + J \frac{h}{d}, \quad h < \frac{6d}{\gamma_s/\tau(h) + J};\]

\[\lambda(h) = 9, \quad h \geq \frac{6d}{\gamma_s/\tau(h) + J};\]

where \(\sigma_z\) is the overburden pressure and \(\gamma_s = \rho_s g\) is the specific weight of the soil. The value of parameter \(J\) in Eq. (9) must be determined empirically. In the absence of such data, the recommended value of \(J = 0.5\) may be used.

### 4. Numerical study with nonlinear soil modeling

The cable-stayed bridge is isolated by using high-damping rubber bearings and the effects of SSI are investigated by performing seismic analysis in time domain using direct integration method. Influence of SSI on the

<table>
<thead>
<tr>
<th>Depth, (h) (m)</th>
<th>Shear modulus, (G_s) (MN/m²)</th>
<th>Mass density, (\rho_s) (Mg/m³)</th>
<th>Poisson’s ratio ((\mu))</th>
<th>Damping ratio ((\xi_s))</th>
<th>Shear strength, (\tau_s) (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>80</td>
<td>400</td>
<td>900</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>125</td>
<td>625</td>
<td>1350</td>
<td>2</td>
<td>2.1</td>
</tr>
<tr>
<td>10</td>
<td>245</td>
<td>1225</td>
<td>2550</td>
<td>2</td>
<td>2.2</td>
</tr>
<tr>
<td>20</td>
<td>550</td>
<td>2750</td>
<td>6500</td>
<td>2.2</td>
<td>2.2</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The seismic response of isolated cable-stayed bridges is investigated under four real strong earthquake ground motions, namely, (1) 1940 Imperial Valley, (2) 1995 Kobe, (3) 1989 Loma Prieta and (4) 1994 Northridge earthquakes. The first one is widely used by the researcher in the past and the last three represent the strong motion earthquake records. The peak accelerations of these earthquake ground motions are shown in Table 4. The specific components of these ground motions applied in the longitudinal and transverse directions are also indicated in Table 4. The displacement and acceleration response spectra of the above four ground motions for 2% of the critical damping are shown in Fig. 3. The maximum ordinates of the spectral acceleration along the longitudinal direction are 1.302 g, 4.12 g, 3.55 g and 3.24 g occurring at the period of 0.46, 0.35, 0.64 and 0.35 s and along the transverse directions are 0.955 g, 3.559 g, 2.215 g and 1.967 g occurring at the period of 0.53, 0.39, 0.4 and 0.51 s for Imperial Valley, Kobe, Loma Prieta and Northridge earthquakes, respectively. The spectra of these ground motion indicate that the ground motions are recorded at hard soil or rock site.

As damping in the structure is very low, 2% damping is assumed. For the HDRB, damping ratio ($\zeta_b$) is taken equal to 10%. Previous studies on seismic isolation of cable-stayed bridges [6,34] have shown that a small amount of isolation is sufficient to reduce the seismically induced forces in the bridge. Higher amount of isolation results in larger displacement of the deck and trade-off to reduction of the base shear of the towers. Hence, herein the isolation period is taken equal to 2 s and is kept constant throughout the study. To evaluate the effects of SSI on the seismic response of the bridge, the evaluation criteria proposed are relative displacement in the longitudinal ($x_{b1}$) and transverse ($y_{b1}$) directions of the isolators at the abutment, relative displacement of the isolators in the transverse direction.

### Table 4

Peak ground acceleration of various earthquake ground motions

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Recording station</th>
<th>Applied in longitudinal direction of the bridge</th>
<th>Applied in transverse direction of the bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Component                      PGA (g)</td>
<td>Component                      PGA (g)</td>
</tr>
<tr>
<td>Imperial Valley, 1940</td>
<td>El Centro</td>
<td>N00E</td>
<td>0.348</td>
</tr>
<tr>
<td>Kobe, 1995</td>
<td>Japan Meteorological Agency</td>
<td>N00E</td>
<td>0.834</td>
</tr>
<tr>
<td>Loma Prieta, 1989</td>
<td>Los Gatos Presentation Center</td>
<td>N00E</td>
<td>0.570</td>
</tr>
<tr>
<td>Northridge, 1994</td>
<td>Sylmar Converter Station</td>
<td>N00E</td>
<td>0.843</td>
</tr>
</tbody>
</table>

Note: PGA = peak ground acceleration.

Fig. 3. Displacement and acceleration spectra of four earthquake ground motions applied in the longitudinal and transverse directions of the bridge.
direction at the deck–tower junction \( (y_{b3}) \), absolute acceleration of the deck in the longitudinal direction at the left abutment \( (x_d) \) and base shear response of the towers along longitudinal \( (V_x) \) and transverse \( (V_y) \) directions at the left tower normalized to the weight of the deck, \( W_d \). Because of the symmetrical structure with anti-symmetry earthquake loading about vertical axis of symmetry, the results at the symmetrical nodes of the structure are found similar in magnitude and direction. Hence, results of the response quantities are presented for left half part of the bridge.

Results of time history analyses under 1940 Imperial Valley earthquake ground motion along longitudinal direction are presented in Figs. 4–6 for soft, medium and firm soil, respectively. The results obtained with and without SSI are plotted in the same graphs for comparison purpose. The trend of the results indicates that there is not much variation in acceleration and base shear response with the type of soil considered. However, for the soft soil strata, little increase in bearing displacement is observed and the response reduces as soil becomes stiffer. Time history plots under 1940 Imperial Valley earthquake ground motion along transverse direction are presented in Figs. 7–9 for soft, medium and firm soil, respectively. It is observed that in transverse direction the deck acceleration and tower base shear found to be increased for soft soil, and the responses reduce as the soil becomes stiffer. However, increase in displacement response of the isolator at the abutment is insignificant when SSI is considered. The dynamic force–displacement loops in longitudinal as well as transverse directions for the soil spring near pile head under the 1995 Kobe earthquake ground motions are presented in Fig. 10. It is to be noted that for the soft soil strata, the displacement response is large and spring force is less. As the soil stiffness increases, the inclination of the loop also increases with increase in spring force and reduction in displacement as expected. Thus, the plots indicate that, by using the Bouc–Wen model for hysteresis systems, the nonlinear behavior of soil strata is adequately captured.

The peak values of the response quantities under different earthquake ground motions are presented in Table 5. It can be observed from the table that the response quantities other than tower base shear in transverse direction are not much affected by the SSI consideration. Since the deck is isolated from the tower, the increase in displacement response due to SSI is little and is in the range of 3–20% in the longitudinal direction, and 3–13% in the transverse direction with soft soil strata. With the firm soil strata, the increase in displacement response of the deck reduces to 10%. Also, it can be observed that, for all the earthquakes except Kobe, 1995, the displacement response of the isolator decreases as the soil stiffness increases. This is in agreement with the trend in displacement spectra of the earthquake ground motions (Fig. 3). Further, it is to be noted from Table 5 that the base shear response of the tower along longitudinal direction is not much influenced by the consideration of SSI. The reason behind this could be explained with the help of earthquake acceleration spectra presented in Fig. 3. The time period of the fixed base tower (i.e. without SSI) along the longitudinal direction comes out to be around 1.7 s and the inclusion

![Fig. 4. Time variation of deck acceleration, tower base shear and bearing displacement in the longitudinal direction of the bridge for soft soil under Imperial Valley, 1940, earthquake motion.](image-url)
of SSI further makes it more flexible. Thus, the time period of the tower with SSI consideration falls in the zone of more or less steady spectral acceleration resulting in insignificant influence on the base shear response.

However, the tower base shear in the transverse direction is greatly affected by the type of soil strata considered. The increase in the tower base shear is about 30–65% for soft site, and gradually approaches to the values

Fig. 5. Time variation of deck acceleration, tower base shear and bearing displacement in the longitudinal direction of the bridge for medium soil under Imperial Valley, 1940, earthquake motion.

Fig. 6. Time variation of deck acceleration, tower base shear and bearing displacement in the longitudinal direction of the bridge for firm soil under Imperial Valley, 1940, earthquake motion.
of the base shear without SSI consideration as the stiffness of the soil increases. This happens because the time period of tower when fixed base is considered comes out to be equal to 0.25 s, which falls in the left side of the peak of acceleration spectra (Fig. 3). When SSI is considered, the tower becomes little flexible and the time period falls in the peak range of the acceleration spectra, which makes the base shear response to increase significantly.

Fig. 7. Time variation of deck acceleration, tower base shear and bearing displacement in the transverse direction of the bridge for soft soil under Imperial Valley, 1940, earthquake motion.

Fig. 8. Time variation of deck acceleration, tower base shear and bearing displacement in the transverse direction of the bridge for medium soil under Imperial Valley, 1940, earthquake motion.
As the soil stiffness increases, the tower base shear in transverse direction approaches to the value of shear without considering the SSI effects (i.e. the fixed base tower case).

5. Numerical study with linear soil modeling

In the practical design of bridges, when it becomes necessary to conduct dynamic response analysis of bridges,
the soil is modeled with linear spring and dashpot system. 
Hence, here in this study, it has been tried to investigate the effectiveness of linear soil model in seismic response of isolated cable-stayed bridge. The peak values of the response quantities assuming completely linear soil model are presented in Table 6. Similar to the nonlinear case, here also the response quantity found affected the most is the tower base shear in the transverse direction. The results show that no convergence (i.e. approaching the response with SSI to that of without SSI) in base shear response is observed with increase in soil stiffness. To visualize the difference in results obtained with the linear and nonlinear soil models, the tower base shear in longitudinal and transverse directions for firm soil are plotted in Fig. 11 in the form of bar chart for all the four earthquakes. The numbers on x-axis show the sequence of earthquakes
considered in this study. The large difference between the results with nonlinear and linear soil models specifically for shear in transverse direction indicates that the consideration of nonlinearity in soil is essential to predict accurate results.

6. Conclusions

Influence of SSI on seismic response of isolated cable-stayed bridge has been investigated under bi-directional earthquake excitations. The deck of the bridge is isolated from the towers by using conventional high-damping rubber bearings. Three types of layered soil strata, namely, soft, medium and firm, have been considered for the study. The soil–pile interaction is idealized as a BNWF using continuously distributed nonlinear springs and viscous dashpots placed in parallel, and Bouc–Wen model is used to model hysteretic behavior of the soil. From the trends of the results of the present study, following conclusions may be drawn:

(1) For soft soil condition, the bearing displacement may be underestimated if SSI is ignored, especially in the longitudinal direction. However, no significant increase takes place in deck displacement in transverse direction.

(2) The tower base shear response in the longitudinal direction is not much affected irrespective of the soil types.

(3) Significant influence of soil–pile interaction is observed on tower base shear response in the transverse direction. The response is much higher as compared to that of the bridge with fixed tower base when the soil is soft to medium. As the stiffness of the soil strata increases the effect of SSI diminishes.

(4) Analyzing the structure assuming linear soil model does not lead to accurate prediction of tower base shear response, and nonlinear soil modeling is essential to reflect dynamic behavior of the soil–pile system properly.

(5) Inclusion of SSI is essential for effective design of seismically isolated cable-stayed bridge, specifically when the towers are very much rigid and the soil condition is soft to medium.

References